* * * Bearing Capacity

The supporting power of a soil (or) Rock is called as Bearing capacity of the soil.

The min^m gross pressure intensity at the base of the footing on which the soil fails in shear is known as UBC (Ultimate Bearing capacity) of the soil. Gross pressure intensity indicates above pressure due to superstructure load, self wt of foundation & soil fill (if any)

The symbol of UBC is q_u (or) q_f
failure

The min^m net pressure intensity at the Base of the footing on which the soil fails in shear, is known as Net UBC

It symbol is q_{nu} (or) q_{nf}

$$\boxed{q_{nu} = q_u - \gamma D}$$

where,

γ = unit wt of soil above base of the footing

D = Depth of the foundation

The ratio of Net ultimate bearing capacity to the FOS against shear is called Net safe bearing capacity.

It symbol is q_{ns}

$$q_{ns} = \frac{q_{nu}}{F} = \frac{q_u - \gamma D}{F}$$

The maxm pressure intensity at the Base of the footing on which soil does not fail in shear is called SBC (Safe Bearing Capacity)

It is symbol is q_s

$$\boxed{q_s = q_{ns} + \gamma D = \frac{q_u - \gamma D}{F} + \gamma D}$$

The pressure intensity at the base of the footing on which the soil neither fails in shear nor in settlement is known as Allowable Bearing Capacity

Development of Bearing Capacity of soil:-

(i) Rankine Earth pressure classical theory

(ii) Prantal theory (Mainly for cohesive soil)

(iii) Dr. Karl Terrazahi theory

(iv) Skempton theory (Mainly for pure cohesive soil $\phi = 0$)

(v) Mr. Meyorhaif theory (improvement on Dr. Terrazahi theory)

(vi) Brinch Hansen theory (Based on Terrazahi theory or improvement of Meyorhaif theory)

(vii) Mr. Vesic theory (Similar to Hansen) (ISI theory)

Dr. Terrazaghi Bearing Capacity theory :-

Dr. Terrazahi made following assumptions :-

I) The soil mass is homogenious, isotrophic & semiinfinite

II) The foundation is shallow one

III) The foundation is of strip type (continuous)

IV) The Base of footing is rough

V) The shear strength theory givens by columb is holds good.

VI) The analysis is of two dimensional

VII) The super position method holds good.

VIII) The triangular soil wedge makes an angle ϕ with horizon

IX) The triangular soil wedge behaves like part of foundation

X) The triangular soil wedge below the Base of footing is decignated as zone-I which is in elastic equillibrium

XI) The zone II is of radial shear zone having Logrithmic spiral curve

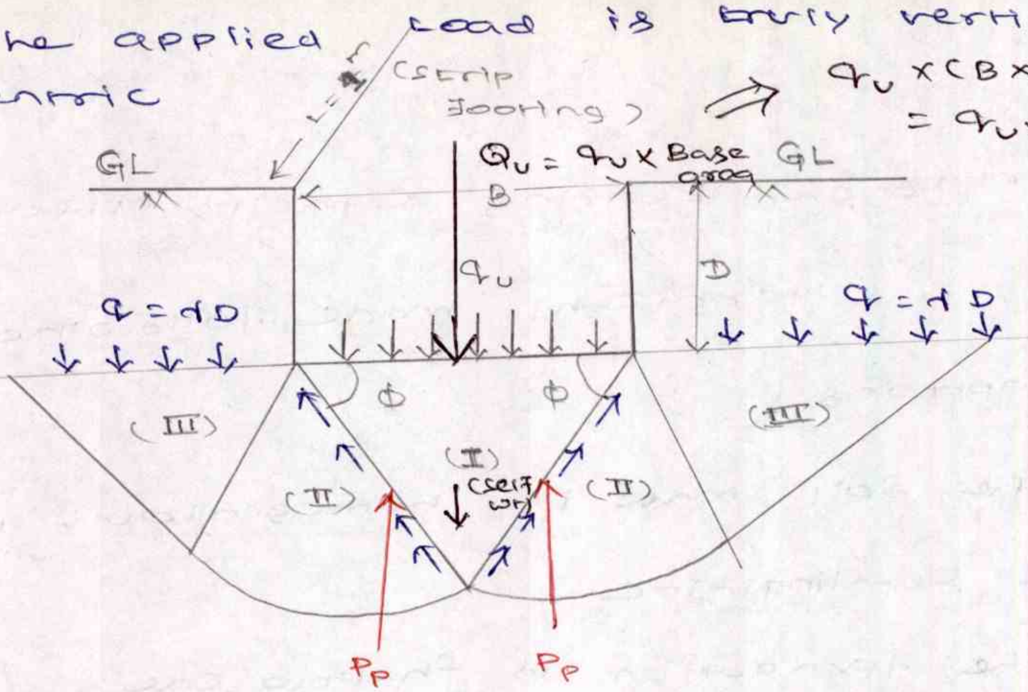
XII) The Zone III is called rankine passive pressure zone or Linear shear zone.

XIII) soil is in dry condition

XIV) The shear failure is of general shear failure

(15) The applied load is truly vertical & concentric

$\Rightarrow q_u \times (B \times 1) = q_u \times B\ (\downarrow)$



Consider elastic equillibrium of triangular soil wedge which is subjected to following types of forces.

(1) Downward vertical load from superstructure including selfwt of footing & soil fill

$$Q_u = q_u \times (B \times 1) \quad (\downarrow)$$

(2) Self wt of triangular soil wedge acting downward

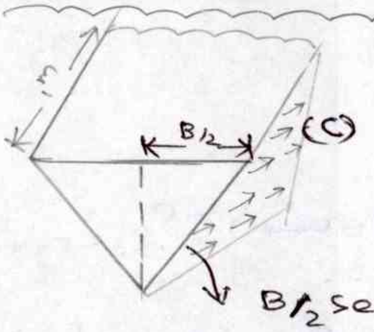
$$y = B/_2 \times \tan\phi$$

$$\text{Self wt} = \gamma \times \text{volume}$$

$$= \gamma \times 1/_2 \times B \times B/_2 \tan\phi \times 1$$

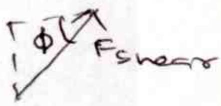
$$W_s = \frac{\gamma B^2}{4} \tan\phi \quad (\downarrow)$$

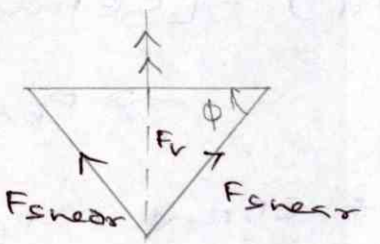
(3) Shear strength of soil :-



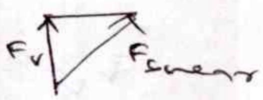
$$F_{shear} = c \times \text{Surface area}$$

$$= c \times (1 \times B/_2 \sec\phi)$$





$$\sin\phi = \frac{F_v}{F_{shear}}$$



$$\Rightarrow F_v = F_{shear} \sin\phi$$

$$F_v = \left(\frac{CB}{2} \sec\phi\right) \sin\phi$$

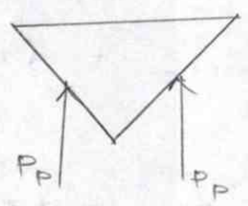
$\uparrow \frac{CB}{2} \tan\phi$

$\uparrow \frac{CB}{2} \tan\phi$

$\uparrow$

$= CB \tan\phi$

(4) Passive Resistance (P_p)



Passive Resistance developes due to three parameters

(i) cohesion of the soil

(ii) surcharge pressure (overburden soil) at the base of the footing (due to γD)

(iii) Due to self wt of soil in linear shear zone

Applying Law of equillibrium,

$\Sigma V = 0$

$$Q_u + W_{self} - F_{s(total)} - P_{p(total)} = 0$$

$$q_u \cdot B + \frac{\gamma B^2}{4}\tan\phi - CB\tan\phi - 2\left[P_{pc} + P_{pq} + P_{p\gamma}\right] = 0$$

$$q_u \cdot B + \left[\frac{\gamma B^2}{4}\tan\phi - 2P_{p\gamma}\right] - \left[CB\tan\phi + 2P_{pc}\right] - 2P_{pq} = 0$$

$$q_u \cdot B = 2P_{p\gamma} - \frac{\gamma B^2}{4}\tan\phi + CB\tan\phi + 2P_{pc} + 2P_{pq}$$

where,

$$2P_{p\gamma} - \frac{\gamma B^2}{4}\tan\phi = \frac{1}{2}\gamma B^2 N_\gamma$$

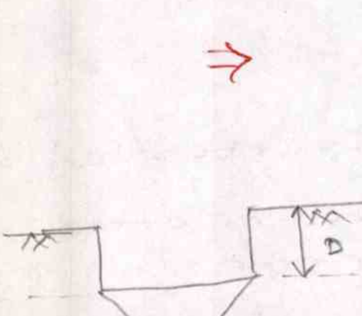
$$CB\tan\phi + 2P_{pc} = CBN_c$$

$$2P_{pq} = (\gamma D)BN_q$$

$$q_u \cdot B = \frac{1}{2}\gamma B^2 N_\gamma + CBN_c + \gamma DBN_q$$

$$\boxed{q_u = \frac{1}{2}\gamma BN_\gamma + cN_c + \gamma DN_q}$$

$$\Rightarrow q_u = \underbrace{cN_c}_{\text{Always Below Base of footing}} + \underbrace{0.5\gamma BN_\gamma}_{\text{Below the base of footing}} + \overbrace{\gamma DN_q}^{\text{Above the Base of footing}}$$



where,

N_c, N_q, N_γ are bearing capacity factors which are dependent on angle of repose (ϕ) only.

Above formula holds good only for strip footing subjected to general shear failure.

But, in case of local shear failure above formula is to be corrected as given below

(i) The value of cohesion is taken as $2/3\,c$

$$c_m = c' = 2/3\,c$$

(ii) The eff. angle of repose is taken as

$$\tan\phi' = 2/3 \tan\phi$$

$$\text{(mobilised) } \phi_m = \phi' = \tan^{-1}(2/3 \tan\phi)$$

(iii) The values of bearing capacity factors are determined with the help of ϕ' only

$$\boxed{q_u = 2/3\,c N_c' + 0.5\gamma B N_\gamma' + \gamma D N_q'}$$

NOTE: 01

(1) There are mainly two type of shear failure as per Dr. Terzaghi theory

a) General shear failure

b) Local shear failure

(2) In case of general shear failure bulging of soil takes place

(3) In case of local shear failure first compn of soil takes place

(4) in case of general shear failure the angle of repose $\phi \geq 38°$

But in case of local shear failure the value of $\phi \leq 28°$

(5) in case of general shear failure the strain developed in the soil is less than 5%.

But in case of local shear failure strain = 10 - 20%.

(6) in case of general shear failure the value of I_D is more than 70%.

But in case of local shear failure the value of I_D is less than 20%.

(7) in case of general shear failure the std penetration number (N is more than (or) equal to 30)

But in case of local shear failure $N < 5$

(8) in case of general shear failure the value of unconfined compressive strength of clay is $\geq$ 100 kPa

But for local shear failure < 100 kPa

Correction in Terzaghi formula of Bearing capacity :-

(I) Size & Shape of the footing :-

General shear failure

Square footing

$$q_u = 1.3\,cN_c + 0.4\,\gamma B N_\gamma + \gamma D N_q$$

Circular footing

$$q_u = 1.3\,cN_c + 0.3\,\gamma B N_\gamma + \gamma D N_q$$

(B → Dia)

Rectangular footing (Raft)

$$q_u = \left[1 + 0.3\frac{B}{L}\right] cN_c + \left[1 - 0.2\frac{B}{L}\right] 0.5\,\gamma B N_\gamma + \gamma D N_q$$

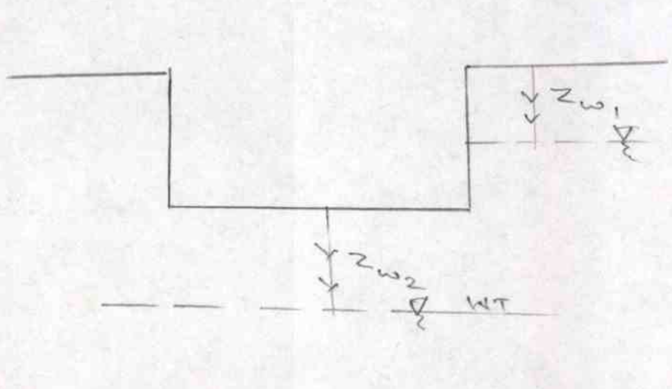
For local shear failure

$$q_u = 1.3\left(\tfrac{2}{3}c\right) N_c' + 0.4\,\gamma B N_\gamma' + \gamma D N_q' \Rightarrow \text{Square}$$

$$q_u = 1.3\left(\tfrac{2}{3}c\right) N_c' + 0.3\,\gamma B N_\gamma' + \gamma D N_q' \Rightarrow \text{circular}$$

$$q_u = \left(1 + 0.3B/L\right)\left(\tfrac{2c}{3}\right) N_c' + \left[1 - 0.2B/L\right] 0.5\,\gamma B N_\gamma' + \gamma D N_q' \Rightarrow \text{Rectangular}$$

(II) Due to presence of water table :-



Due to rise of water table the unit wt of soil comes down (γ_{sub})

Generally the value of $\gamma_{sub} = 50\% \ \gamma_{sat}$

$$\gamma_{sub} = \gamma_{sat} - \gamma_{w}$$
$$= 19.8 - 9.81$$
$$\simeq \tfrac{1}{2} \times 19.8$$
$$\simeq \tfrac{1}{2} \times \gamma_{sat}$$

If the soil below GL is sandy (cohesionless) then the value of UBC reduces by 50%. (Reduced to)

According to Dr. Terrazahi UBC can be corrected as below

$$q_u = CN_c + 0.5\gamma B N_\gamma R_{w_2} + \gamma D N_q \cdot R_{w_1}$$

where,

R_{w_1} = Reduction factor due to presence of water table

$$R_{w_1} = 0.5\left[1 + \frac{z_{w_1}}{D}\right]$$

R_{w_2} = Reduction factor

$$R_{w_2} = 0.5\left[1 + \frac{z_{w_2}}{B}\right]$$

Case: 01

WT at GL

$$Z_{w_1} = 0 \;; \quad Z_{w_2} = 0$$

$$R_{w_1} = 0.5\left[1 + \frac{Z_{w_1}}{D}\right] = 0.5$$

$$R_{w_2} = 0.5\left[1 + \frac{Z_{w_2}}{B}\right] = 0.5$$

Case: 02

WT at the base of the footing

$$Z_{w_1} = D \;; \quad Z_{w_2} = 0$$

$$R_{w_1} = 0.5\left[1 + \frac{Z_{w_1}}{D}\right] = 1$$

$$R_{w_2} = 0.5\left[1 + \frac{Z_{w_2}}{B}\right] = 0.5$$

Case: 03

WT is at far away from the base of the footing

$$Z_{w_2} \geq B$$

$$Z_{w_1} = D$$

$$R_{w_1} = 0.5\left[1 + \frac{Z_{w_1}}{D}\right] = 1$$

$$R_{w_2} = 0.5\left[1 + \frac{Z_{w_2}}{B}\right] = 1$$

Note: 01

In case of pure cohesive soil $\phi = 0$

$$N_c = 5.7$$

Skempton

$$N_q = 1$$

$$N_\gamma = 0$$

$$\therefore q_u = cN_c + 0.5\gamma B N_\gamma + \gamma D N_q$$

$$\boxed{q_u = 5.7c + \gamma D}$$

Note: 02

If the footing is at GL

then $D = 0$

and hence for pure cohesive soil ($\phi = 0$)

$$\boxed{q_u = 5.7\,c}$$

It means the Bearing capacity of pure cohesive soil resting on GL is independent of size of footing.

Note: 03

In case of cohesionless soil the bearing capacity is dependent on size of the footing

$c = 0$

$$q_u = cN_c + 0.5\gamma B N_\gamma + \gamma D N_q$$

$$q_u = 0 + 0.5\gamma B N_\gamma + \gamma D N_q$$

(at the GL)

Equal size

$$\frac{q_u\,(\text{strip})}{q_u\,(\text{square})} = \frac{0.5\gamma B N_\gamma}{0.4\gamma B N_\gamma} = \frac{0.5}{0.4} \simeq 1.25$$

(At the GL)

$$\frac{q_u\,(\text{strip})}{q_u\,(\text{square})} = \frac{0.5\gamma B_1 N_\gamma}{0.4\gamma B_2 N_\gamma} = 1.25 B_1/B_2$$

Note: 04

$$q_{nu} = q_u - \gamma D$$

$$= cN_c + 0.5\gamma B N_q + \gamma D N_q - \gamma D$$

$$q_{nu} = cN_c + 0.5\gamma B N_q + \gamma D (N_q - 1)$$

Skempton's theory :-

According to Dr. Terzaghi the value of N_c is independent of depth. which is not correct as per Mr. Skempton

For pure Cohesive soil ($\phi=0$), Only Skempton theory is adopted (Better then Dr. Terzaghi theory)

According to Mr. Skempton, the value of N_c is not constant but varies as per D/B ratio

① For strip footing

$$N_c = 5.14 \simeq 5 \; ; \; (D=0)$$

$$N_c = 5\left[1+0.2\frac{D}{B}\right] \; ; \; D/B < 2.5$$

$$N_c = 1.5\times 5 \simeq 7.5 \; ; \; D/B > 2.5$$

(ii) For circular & square footing

$$N_c = 6 \; ; \; D/B = 0$$

$$N_c = 6\left[1+0.2D/B\right] \; ; \; D/B < 2.5$$

$$N_c = 1.5\times 6 = 9 \; ; \; D/B > 2.5$$

Mainly for pile foundation

(iii) $N_c = \left[1+0.2\frac{B}{L}\right]\times N_{c\,strip}$; $D/B = 0$

$N_c = \left[1+0.2\frac{B}{L}\right]\times N_{c\,(strip)}$; $D/B < 2.5$

$N_c = \left[1+0.2\frac{B}{L}\right]\times N_{c\,(strip)}$; $D/B > 2.5$

Numericals :-

① A shallow found^n having size $2 \times 2m$ is founded at a depth of 1.5^m below GL. The soil properties above the base of the footing are $C = 15$ kPa ; $\gamma = 17$ kN/m^3 ; The properties of soil below the base of the footing are $c = 0$; $\phi = 38°$; $\gamma = 19.5$ kN/m^3. The water table is at 1m below GL. Take, $N_c = 34$; $N_\gamma = 33$; $N_q = 25$.

Determine

(i) UBC

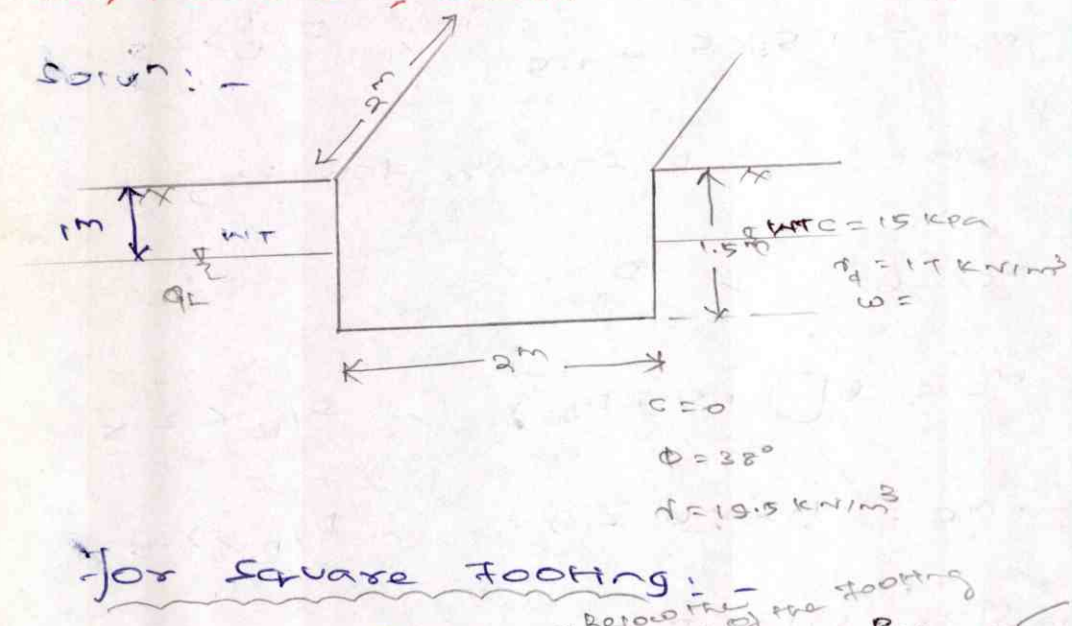
(ii) ~~f~~BC

(iii) safe load on the footing

(iv) safe load on the footing if eccentricity in x-direction = 0.4 ; y-direction = 0.4

Take, FOS = 3 ; water content = 25%.

Soln :-



$\gamma_b = \gamma_d (1 + w)$

$\gamma_{bulk} = 21.25$ kN/m^3

For square footing :-

Below the base of the footing — surcharge pressure

$$q_u = 1.3 C N_c + 0.4 \gamma B N_\gamma R_{w_2} + \gamma D N_q R_{w_1}$$

Below the Base — Above the base

$= 1.3 \times 0 + 0.4 \times 19.5 \times 2 \times 33 \times 0.5 [1 + 0]$ + $\cancel{21.25 \times 1.5}$ 22.75 $\times 25 \times 0.5 [1 + \frac{1}{1.5}]$

0.5

0.833

$$\boxed{q_u = 826.15 \text{ kN/m}^3}$$

R_{w_1}

$\gamma D = \sigma' = 17 \times 1 + 21.25 \times 0.5 - 9.81 \times 0.5$

$= 22.75$ kN/m^2

$$q_s = q_{ns} + \gamma D$$

$$= \frac{q_u - \gamma D}{F} + \gamma D$$

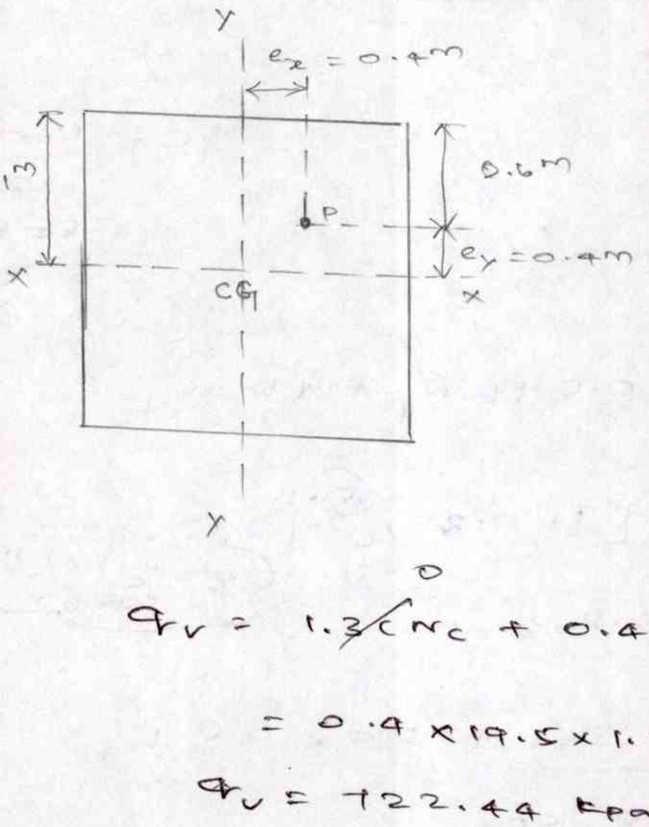
$$= \frac{826.15 - 22.75}{3} + 22.75$$

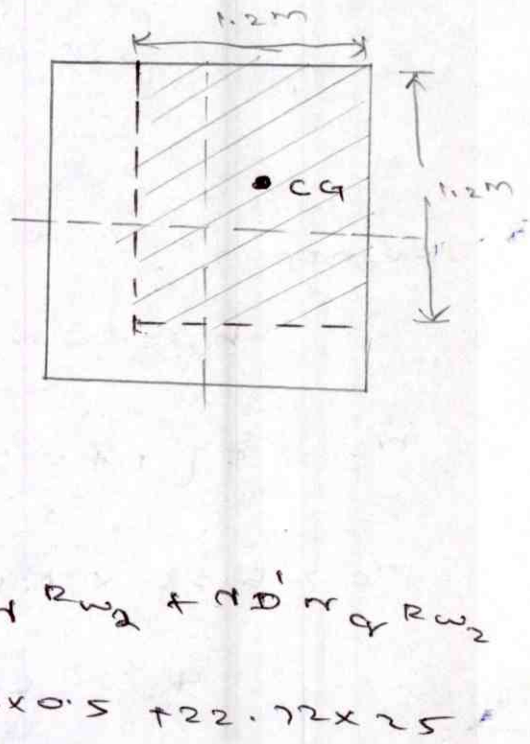
$$\boxed{q_s = 290.55 \ kN/m^2}$$

Q_s (safe Load) $= q_s \times (L \times B)$

$$= 290.55 \times 2 \times 2$$

$$\boxed{Q_s = 1162.2 \ kN}$$





$$q_u = 1.3\cancel{c}N_c + 0.4\gamma B' N_\gamma R_{w_2} + \gamma D' N_q R_{w_2}$$

$$= 0.4 \times 19.5 \times 1.2 \times 33 \times 0.5 + 22.72 \times 25$$

$$q_u = 722.44 \ kPa$$

$$q_s = \frac{q_u - \gamma D}{F} + \gamma D = \frac{722.44 - 22.72}{3} + 22.72$$

$$q_s = 255.96 \ kPa$$

$$Q_s = q_s \times (B' \times L')$$

$$= 255.96 \ (1.2 \times 1.2)$$

$$\boxed{Q_s = 368.58 \ kN}$$

② A Raft foundation having size $6^m \times 12^m$ is founded at a depth $2m$ below the GL. The soil properties are $\phi = 0$; Unconfined compressive strength $= 120$ kPa

Determine i) UBC

ii) SBC

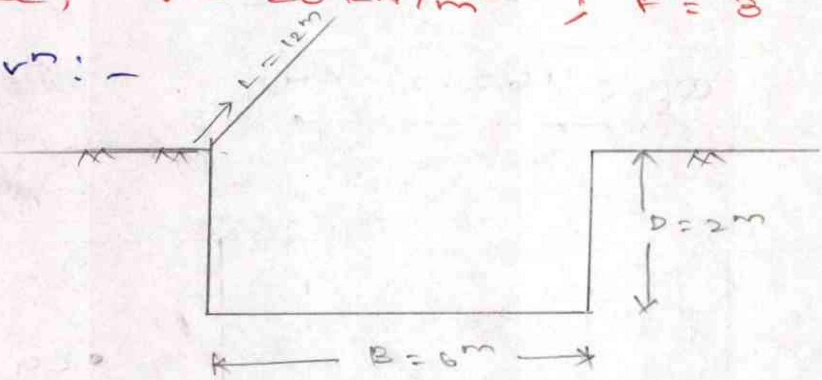
iii) Safe Load

use Skempton theory

what will be the difference in safe load if Terzaghi theory is adopted.

Take, $\gamma = 20\ kN/m^3$; $F = 3$

Soln:-



$C = \frac{120}{2}$

$= 60$ kPa

Skempton ;

$$q_u = cN_c + \cancel{0.5\gamma B N_\gamma}^{\,0} + \gamma D N_q^{\,\nearrow 1}$$

$$N_c = 5\left[1 + 0.2\frac{D}{B}\right]\left[1 + 0.2B/L\right]$$

$$\left\{ N_c = N_{c\,(strip)} \times (1 + 0.2B/L) \right\}$$

$$N_c = 5.33 \times 1.1 = 5.863$$

$$q_u = 60 \times 5.863 + 20 \times 2 \times 1$$

i) UBC $\boxed{q_u = 391.78\ \text{kPa}}$

ii) SBC $q_s = \frac{q_u - \gamma D}{F} + \gamma D = \boxed{157.26\ \text{kPa}}$

iii) $Q_s = q_s \times B \times L$

$= 157.26 \times 6 \times 12$

Safe Load $\boxed{Q_s = 11322.72\ \text{kN}}$

For Terrazaghi

$$q_u = cN_c\left[1+0.3\frac{B}{L}\right] + 0.5\cancel{\gamma B}N_\gamma\left[1+0.2B/L\right] + \gamma D N_q$$

$$= 60\times 5.7^{\times 1.15} + 20\times 2\times 1$$

$$q_u = 438.3 \text{ kPa}$$

$$q_s = \frac{q_u - \gamma D}{F} + \gamma D$$

$$q_s = 17[illegible] \text{ kPa}$$

$$Q_s = q_s \times B \times L$$

$$\boxed{Q_s = 12319.2 \text{ kN}}$$

$q_g = \frac{q_u}{F}$

Gross pressure

Diff. in safe Load

$$12319.2 - 11322.72$$

$$= \cancel{11322.72 - 11058}$$

$$= \cancel{234} \quad \boxed{996.48 \text{ kN}}$$

Ans

---

③ Two circular footing of diameters D_1 & D_2 are resting on the surface of the same purely cohesive soil. Determine the ratio of their UBC $\phi = 0$

Soln :-

$$q_u = 1.3cN_c + 0.3\gamma B N_\gamma + \gamma D N_q$$

$$q_{u_1} = 1.3c\times 5.7$$

$$q_{u_2} = 1.3c\times 5.7$$

$$\frac{q_{u_1}}{q_{u_2}} = \boxed{1}$$

Ans

(4) A strip footing 8m wide is designed for a total settlement of 40mm. ×

(4) A footing 2.25m square is located at 1.5m in a sand of unit wt 18 kN/m³. Take, c = 0 ; ϕ = 36° calculate the safe load. Take Terrazagni bearing capacity factors $N_c = 65.4$; $N_\gamma = 49.4$; $N_q = 54$

Take, FOS = 8

Soln :-

$$q_u = 1.3\cancel{C}N_c + 0.4\gamma B N_\gamma + \gamma D N_q$$

$$= 0.4 \times 18 \times 2.25 \times \overset{54}{\cancel{49.4}} + 18 \times \overset{1.5}{\cancel{2.25}} \times 49.4$$

$$q_u = \overset{2208.88}{\cancel{2368.728}} \text{ Kpa}$$

$$q_s = \frac{q_u - \gamma D}{F} + \gamma D$$

$$q_s = 754.26 \text{ Kpa}$$

$$Q_s = q_s \times (2.25 \times 2.25)$$

$$\boxed{Q_s = 3818.14 \text{ kN}}$$

Ans

(5) A square footing having size $2^m \times 2^m$ is founded below GL at a depth 1.5m the load is applied at an angle 15° with the vertical on the footing. Soil properties are $c = 0$; $\phi = 38°$; $\gamma = 17.5\ kN/m^3$ WT is at GL take, $\gamma_{sat} = 20\ kN/m^3$. Use BIS method (Bureau of Indian std) Determine net ultimate bearing capacity & safe permissible load on the footing. Take, FOS = 3

Soln :-

Note:-

According to IS6403-1981 the ultimate bearing capacity of soil is given by formula Based on Mr. Vesic

$$\boxed{q_u = cN_c s_c d_c i_c + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma \times R_{w2} + \gamma D N_q s_q d_q i_q \times R_{w1}}$$

Where,

Bearing capacity factors are as below

$N_c = (N_q - 1)\cot\phi$

$N_q = e^{\pi\tan\phi} \times \tan^2(45° + \phi/2)$

$N_\gamma = 2[N_q + 1]\tan\phi$

Shape factor

$s_c = 1 + 0.2B/L$ → Rectangular footing

$= 1.3$ → Circular & square footing

$= 1$ → strip footing

$s_q = 1 + 0.2B/L$ → Rectangular

$= 1.2$ → Circular & square footing

$s_\gamma = 1 - 0.4B/L$ → Rectangular

$= 0.8$ → square

$= 0.6$ → circular

Depth factor:-

$$d_c = 1 + 0.2D/B \tan(45^\circ + \phi/2)$$

$$d_q = d_\gamma = 1 + 0.1D/B \tan(45^\circ + \phi/2) \quad ; \quad \phi > 10^\circ$$

$$d_q = d_\gamma = 1 \quad ; \quad \phi < 10^\circ$$

Inclination factor:-

Degree

$$i_c = i_q = \left[1 - \frac{\alpha}{90}\right]^2$$

$$i_\gamma = \left[1 - \frac{\alpha}{\phi}\right]^2$$

Given data,

$\phi = 38^\circ$ $\alpha = 15^\circ$

$B = 2m$ (square) $c = 0$

$D = 1.5m$

$\gamma_{sat} = 20\ kN/m^3$

$N_q = e^{\pi \tan\phi} \times \tan^2(45^\circ + \phi/2)$

$= 48.933$

$N_c = (N_q - 1)\cot\phi$

$= 61.35$

$N_\gamma = 2(N_q + 1)\tan\phi$

$= 78.024$

$S_c = 1.3$

$S_q = 1.2$

$S_\gamma = 0.8$

$d_c = 1 + 0.2D/B \tan(45^\circ + \phi/2)$

$d_c = 1.3075$

$d_q = d_\gamma = 1 + 0.1D/B \tan(45^\circ + \phi/2)$

$d_q = d_\gamma = 1.1537$

$i_c = i_q = \left[1 - \frac{\alpha}{90}\right]^2 = 0.694$

$i_\gamma = \left[1 - \frac{\alpha}{\phi}\right]^2 = 0.3663$

$q_u = cN_c\, s_c . d_c . i_c + 0.5\gamma B N_\gamma\, s_\gamma d_\gamma i_\gamma R_{w2} + \gamma D N_q\, s_q d_q i_q R_{w1}$

$q_u = \cancel{1038.07}$ 969.007 kN/m^2

$q_c = \frac{q_v - \gamma D}{F} + \gamma D = 383.\ kN/m^2$

$\therefore \; q_{nu} = q_u - \gamma D \times R_{w1}$ $\quad Q_s = 343 \times 2 \times 2$

$= 969 - 20 \times 1.5 \times 0.5$ $\quad = 1332 \text{ kN}$

$q_{nu} = 954 \text{ kN/m}^2$

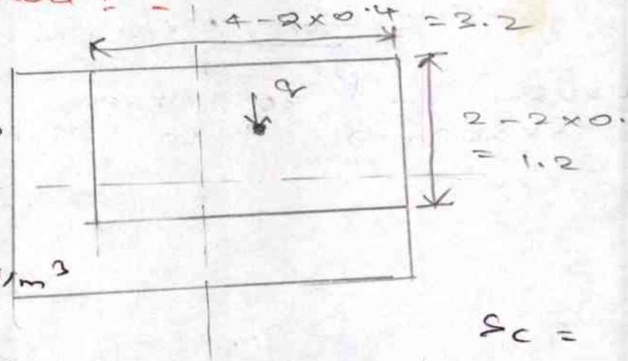
Ans:-

Net ultimate bearing capacity $= 954 \text{ kN/m}^2$

Safe permissible Load $= 1332 \text{ kN}$

---

⑥ A Rectangular footing having size $2^m \times 4^m$ is founded below GL at a depth of 1.5m. The soil properties are $C = 30 \text{ kPa}$; $\phi = 22°$; $\gamma = 17.5 \text{ kN/m}^3$; WT is at far away from the base. The load applied on the footing is inclined at an angle $10°$ with the vertical. There is an eccentricity of 40cm for breadth only. Determine net ultimate bearing capacity. Use BIS method :-



$\phi = 22°$

$D = 1.5 \text{m}$ $\quad e = 0.4^m$

$B = 2^m$ $\quad ; L = 4^m$

$C = 30$ $\quad \gamma = 17.5 \text{ kN/m}^3$

$\alpha = 10°$

$B' = 1.2$

$L' = 3.2$

$N_q = e^{\pi \tan\phi} \times \tan^2(45° + \phi/2)$

$= 7.82$

$N_c = (N_q - 1)\cot\phi$

$= 16.88$

$N_\gamma = 2[N_q + 1]\tan\phi$

$= 7.127$

$R_{w1} = R_{w2} = 1$

$s_c = 1 + 0.2B/L = 1.075$

$s_q = 1 + 0.2B/L = 1.075$

$s_\gamma = 1 - 0.4B/L = 0.85$

$d_c = 1.37$

$d_q = d_\gamma = 1.185$

$i_c = i_q = \left[1 - \frac{\alpha}{90}\right]^2 = 0.79$

$i_\gamma = \left[1 - \frac{\alpha}{\phi}\right]^2 = 0.2975$

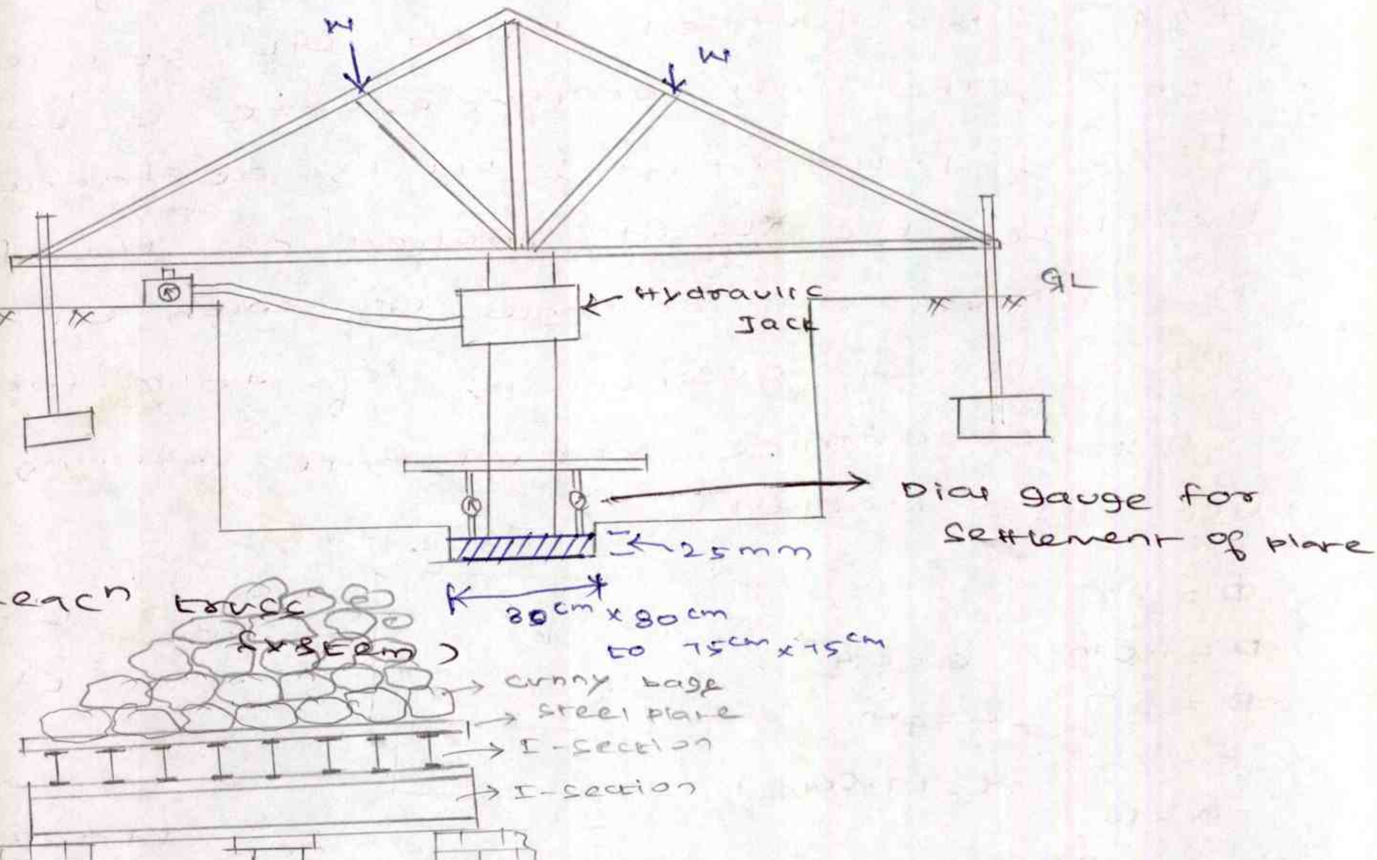
$$q_u = cN_c s_c d_c i_c + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma R_{w_2} + \gamma D N_q s_q d_q i_q R_{w_1}$$

$q_u = 818.187\ kN/m^2$

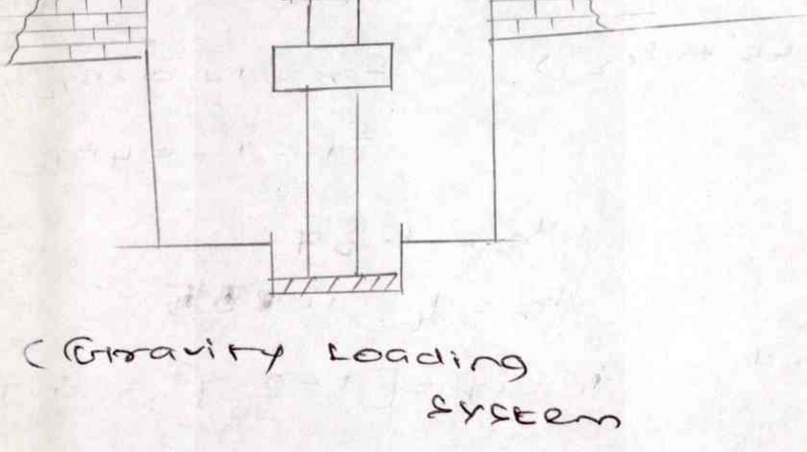
$q_{nu} = q_u - \gamma D$

$q_{nu} = 791.937\ kN/m^2$

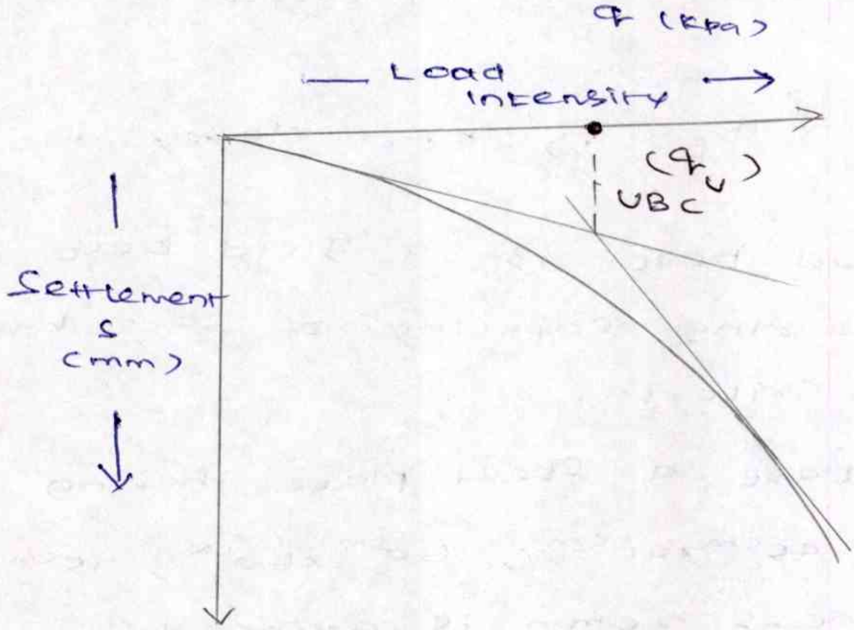
PLATE LOAD TEST

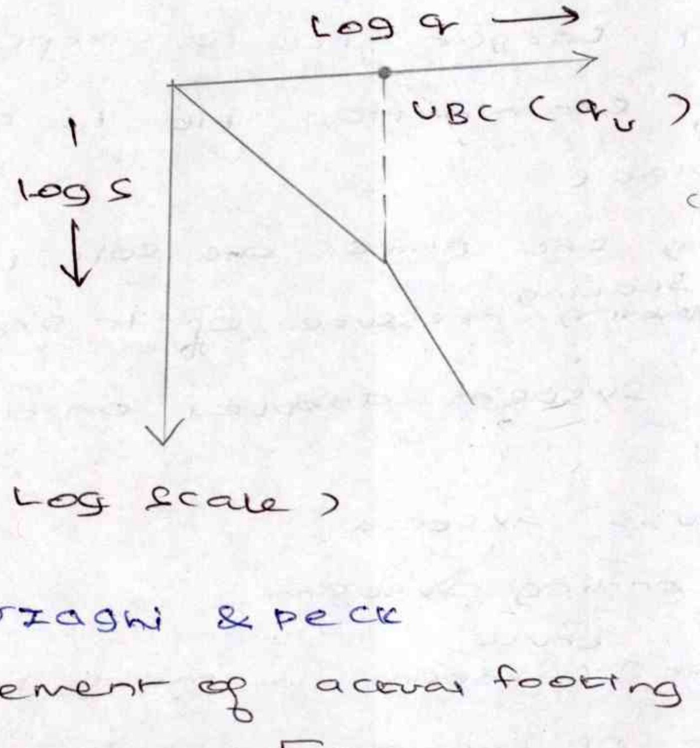


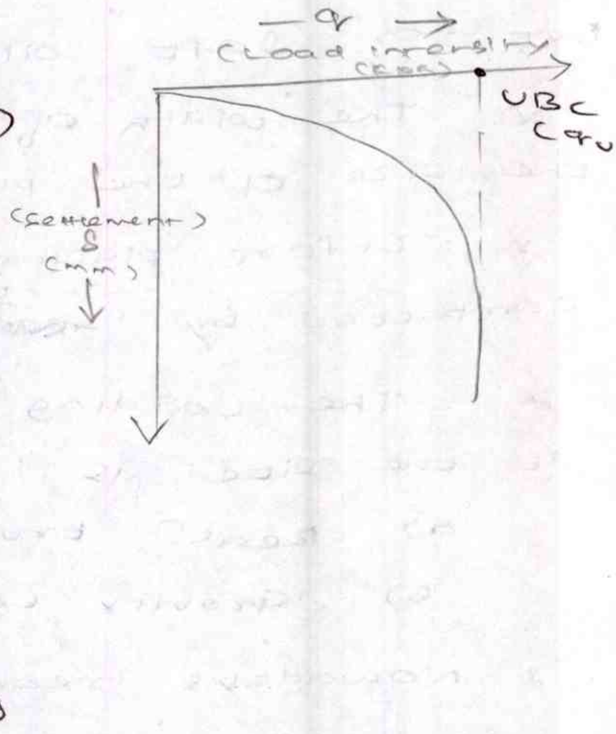
(Reach truss system)



(Gravity loading system)







(Log Log scale)

Terzaghi & Peck

Settlement of actual footing

$$S_f = S_p \left[\frac{B_f (b_p + 0.3)}{b_p (B_f + 0.3)} \right]^2 \Rightarrow \text{Sandy soil (Granular soil)}$$

where,

B_f = width of footing in m

b_p = width of plate in m

$$S_f = S_p \times \frac{B_f}{b_p} \Rightarrow \text{for Clayey soil}$$

$$q_f = q_p \times \frac{B_f}{b_p} \Rightarrow \text{sandy soil (scale effect)}$$

$$q_f \simeq q_p \Rightarrow \text{For clayey soil}$$

* plate Load test is a field test to determine Bearing capacity of soil based on settlement criteria.

* In this test a steel plate having size $30^{cm} \times 30^{cm}$; $45^{cm} \times 45^{cm}$; $60^{cm} \times 60^{cm}$; $75^{cm} \times 75^{cm}$ having thickness 25mm is adopted

* For dense soil smaller size is adopted But for soft soil Larger size is adopted.

* The width of examination pit is 5 times the size of the plate

* Before placing the plate the soil is compacted by ~~[illegible]~~ seating pressure of $70\ g/cm^2$

* The loading system adopted on the plate at the site is
  a) Reac^n truss system
  b) Gravity Loading system

* Nowadays reac^n truss system is more popular

The settlement of the plate is determined with the help of three dial gauges placed at 120° interval.

* The mean of three dial gauges values is the settlement on a perticular intensity of Load

* A graph is plotted in between the settlement of the plate & intensity Load applied on the plate as shown in above fig.

* The value of UBC of the soil is determined from the graph as shown in above fig.

Drawbacks :-

(I) This test is not accurate for clayey soil where the ultimate settlement can not be reached in a limited time

(II) This test is value less for sandy soil where scale effect takes place.
∴ Linear interpolation to be applied to get the bearing capacity of a footing on a sandy soil.

(III) If water table is near GL then this test does not give accurate result

(IV) This test is not accurate if the soil below the foundation (or) test pit is not homogenious for a depth = 2 times width of the footing

Note :-
Housel formula :-

According to Mr. Housel the applied load on a footing is equal to $q \times A + s \times p$

$$\boxed{Q = q \times A + s \times p}$$

where,

q = pressure intensity at the base of the footing

A = Base area of the footing

s = circumferential shear

p = perimeter of the footing

(7) In a plate load test conducted on cohesionless soil a 600 mm square test plate settles by 15 mm determine the settlement of 1m square footing?

$$S_f = S_p \left[\frac{B_f (b + 0.3)}{b [B_f + 0.3)} \right]^2$$

$$= 15 \left[\frac{1 (0.6 + 0.3)}{0.6 (1 + 0.3)} \right]^2$$

$$= 19.97 \text{ mm}$$

$$\boxed{S_f \simeq 20\text{mm}}$$

(8) A square footing having size 1m x 1m carries a load of 150 kN. another footing of size 2m x 2m carries a load of 300 kN determine the load carrying capacity of footing having size 3m x 3m

Soln:-

$$Q_1 = q A_1 + S . x P_1 \quad \text{—— (1)}$$

$$Q_2 = q A_2 + S x P_2 \quad \text{—— (2)}$$

$$Q_3 = q A_3 + S P_3 \quad \text{—— (3)}$$

$$150 = q (1 \times 1) + S (4 \times 1) \quad \text{—— (1)}$$

$$300 = q (2 \times 2) + S (4 \times 2) \quad \text{—— (2)}$$

By solving (1) & (2)

$$q = 0 \; ; \; S = 37.5 \text{ kN/m}$$

$$Q_3 = q A_3 + S P_3$$

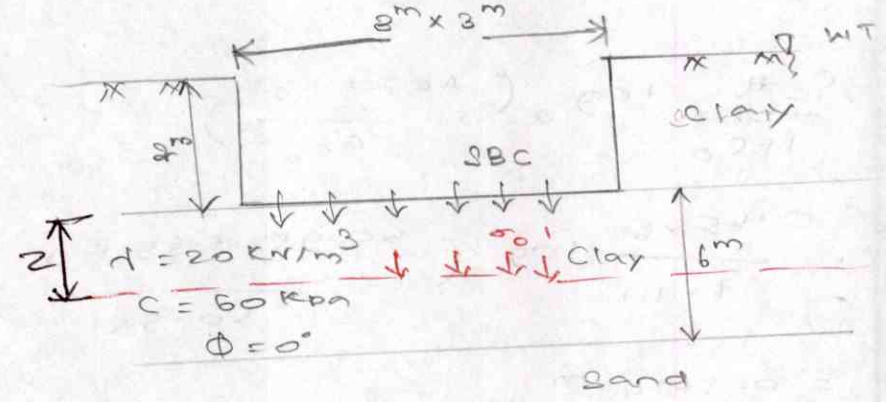
$$= 0 + 37.5 \times (4 \times 3)$$

$$\boxed{Q_3 = 450 \text{ kN}}$$

9) Determine the consolidation settlement of clayey soil having profile as shown in fig below. use 2:1 method & Skempton method take FOS = 3.

D = 2m

B = 3m



LL = 60%

e = 1.1

By Skempton

$$q_u = C N_c + 0.5 \gamma B N_\gamma R_{w2} + \gamma D N_q R_{w1}$$

$D/B < 2.5$

$$N_c = 5[1 + 0.2 D/B]$$

$$= 6.8$$

$N_c = 6.8$

$N_\gamma = 0$

$N_q = 1$

$$= 60 \times 6.8 + 20 \times 2 \times 1 \times 0.5$$

$$q_u = 428 \text{ KN/m}^2$$

$$q_s = \frac{q_u - \gamma D}{F} + \gamma D$$

$$\boxed{q_s = 156 \text{ KN/m}^2}$$

$$S = \frac{C_c \cdot H}{1 + e_0} \log_{10}\left(\frac{\sigma_0' + \Delta\sigma'}{\sigma_0'}\right) = 0.265$$

$C_c = 0.009(LL - 10\%)$

$= 0.45$

$\sigma_0' = \gamma_1 h_1 + \gamma_2 h_2 - \gamma_w h_w$

$= 20 \times 2 + 20 \times 3 - 9.81 \times (3+2)$

$\sigma_0' = 50.95 \text{ KN/m}^2$

$$\Delta\sigma = \frac{Q}{(B+z)(L+z)} = \frac{q_s \times A}{(B+z)^2}$$

$$= \frac{15638 \times 3 \times 3}{(3+3)^2}$$

$$\Delta\sigma = 39.095 \ kN/m^2$$

$$S = \frac{C_c H}{1+e_0} \log_{10}\left(\frac{\Delta\sigma_0 + \Delta\sigma'}{\Delta\sigma_0}\right)$$

$$= \frac{0.45 \times 6}{1+1.1} \log_{10}\left(\frac{50.95 + 39.095}{50.95}\right)$$

$$\boxed{S = 0.3179 \ m}$$

100% primary $\boxed{S_{final} = 31.8 \ cm}$

BEARING CAPACITY

29) Find net SBC of Rectangular footing having size $2^m \times 4^m$ placed at 1^m below GL. The eccentricity of load is 15cm in breath direction only. The inclination of load is $10°$ with the vertical. Take eff. soil properties, $c' = 20$ kpa; $\phi' = 25°$ & $\gamma = 18$ kN/m³. The WT is at the base of the footing. Adopt, ISI method.

From IS: 6403 - 1981

$$q_u = cN_c S_c d_c i_c + 0.5\gamma_B \cdot B N_\gamma S_\gamma \cdot d_\gamma \cdot i_\gamma \, R_{w_2}' + \gamma D_{R_{w_1}} N_q \cdot d_q \cdot i_q \cdot S_q$$

Where,

$N_q = e^{\pi \tan\phi} \cdot \tan^2(\pi/4 + \phi/2)$; S_c, S_γ, S_q - Shape factors

$N_c = (N_q - 1)\cot\phi$; d_c, d_γ, d_q - depth factors

$N_\gamma = 2(N_q + 1)\tan\phi$; i_c, i_γ, i_q - inclination factors

Note:-

1) Dr. Terzaghi did not consider base factor, ground factor, inclination factor, depth factor

2) Mr. Mayerhof considered shape factor, depth factor & inclination factor

3) Mr. Brinch handson considered all possible factors like shape factor, depth factor, inclination factor base factor & ground factor

4) Mr. Vesic adopted all factors given by handson but he change the bearing capacity factor relation

5) IS : 6403-1981 has adopted vesic bearing capacity factors but ground factors & base factors have been neglected.

According to IS : 6403-1981, the relation given as below.

$$q_u = c N_c s_c d_c . i_c + 0.5 \gamma B N_\gamma s_\gamma d_\gamma i_\gamma R_{w_2}$$

$$+ \gamma D R_{w_1} N_q . s_q . d_q . i_q$$

σ'

where,

Bearing capacity factors. (N_c, N_q, N_γ)

$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$ If $\phi = 0$

$N_c = (N_q - 1) \cot\phi$ $N_c = 5.14$

$N_\gamma = 2(N_q + 1) \tan\phi$ $N_q = 1$

$N_\gamma = 0$

Shape factors :-
(s_c, s_q, s_γ)

Shape of footing :	s_c	s_q	s_γ
Strip / wall footing	1.0	1.0	1.0
Rectangle	$(1+0.2B/L)$	$(1+0.2B/L)$	$(1-0.4B/L)$
Square	1.3	1.2	0.8
Circle	1.3	1.2	0.6

Depth factors :-

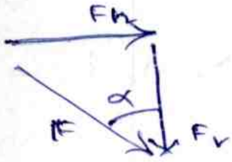
$$d_c = 1 + 0.2 \frac{D_f}{B} \sqrt{N_\phi}$$

$$d_q = d_\gamma = 1 + 0.1 \frac{D_f}{B} \sqrt{N_\phi} \quad \text{For } \phi > 10° \qquad N_\phi = \frac{1+\sin\phi}{1-\sin\phi}$$

$$d_q = d_\gamma = 1 \; ; \quad \text{For } \phi < 10°$$

Load inclination factors :-

$i_c = i_q = \left(1 - \frac{\alpha}{90}\right)^2$



$\alpha = \tan^{-1}(F_h/F_v)$

$i_\gamma = \left(1 - \frac{\alpha}{\phi}\right)^2$

Soln :-

$\Rightarrow$

$N_q = e^{\pi \tan\phi} \left(\tan^2(\pi/4 + \phi/2)\right) = 10.66$

$N_c = (N_q - 1)\cot\phi = 20.715$

$N_\gamma = 2(N_q + 1)\tan\phi = 10.87$

$\Rightarrow$

$S_c = (1 + 0.2 B/L)$

$L = L' = 4\,m$

$B' = B - 2e = 2 - 2(0.15)$

$= 1.7\,m$

$S_c = 1.085$

$S_q = (1 + 0.2 B/L) = 1.085$

$S_\gamma = (1 - 0.4 B/L) = 0.83$

$\Rightarrow$

$d_c = 1 + 0.2 \frac{D_f}{B} \sqrt{N_\phi}$

$N_\phi = \frac{1 + \sin\phi}{1 - \sin\phi} = 2.464$

$d_c = 1.\cancel{1}85$

$d_q = d_\gamma = 1 + 0.\cancel{1} \frac{D_f}{B} \sqrt{N_\phi} = 1.09$

$\Rightarrow$ $i_c = i_q = \left(1 - \frac{\alpha}{90}\right)^2 = 0.79$

$\alpha = 0$

$i_\gamma = \left(1 - \frac{\alpha}{\phi}\right)^2 = 0.36$

$R_{w_1} = 1 ; \; R_{w_2} = 0.5$

$$q_{nu} = cN_c s_c d_c i_c + 0.5\gamma B N_\gamma s_\gamma d_\gamma i_\gamma R_{w_2} + qD R_{w_1} (N_q - 1) d_q i_q s_q$$

$$= 20 \times 20.715 \times 1.085 \times 1.185 \times 0.79 + 0.5 \times 18 \times 1.7 \times 10.87 \times 0.83 \times 1.09 \times 0.36 \times 0.5 + 18 \times 1 \times 1 \times (10.66 - 1) \times 1.09 \times 0.79 \times 1.085$$

$$\boxed{q_u = 627.169 \text{ kPa}}$$

Ans

$q_{nu} = q_u - qD = \boxed{609.17 \text{ kPa}}$ Ans $; \; q_{ns} = \frac{q_{nu}}{F} = 203.1 \text{ kPa}$

$\Rightarrow q_s = \frac{q_{nu}}{F} + qD = \frac{609.17}{3} + 18 \times 1 = 221 \text{ kPa}$

$Q_s = 221 \times 4 \times 1.7 = 1502.8 \text{ kN}$

Ans

Note:-

max^m permissible safe bearing capacity of soil

IS:1904

		SBC of soil
1	very soft clay	- 50 kPa (5 t/m²)
2)	soft clay	- 100 kPa
3)	medium clay	- 150 kPa
4)	Black cotton soil	- 150 kPa
5)	stiff clay	- 250 kPa
6)	very hard clay	- 450 kPa